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TECHNICAL MEMORANDUMS

NATIONAL ADVISORY COMMITTEE FOR AERONAUTICS

No. 332

CONCRETE AIRSHIP SHEDS AT ORLY, FRANCE

By Freyssinet

PART I

GENERAL ASPECT OF THE PROBLEM

BASIC PRINCIPLES OF THE FINAL PROJECT

PRINCIPAL STRUCTURAL ELEMENTS

From "Le Genie Civil," September 22 and 29, 1923.

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NATIONAL ADVISORY COMMITTEE FOR AERONAUTICS.

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CONCRETE AIRSHIP SHEDS AT ORLY, FEANCE.*

By Freyssinet.

PART I.

Contest for the construction of the Orly sheds.—The Under-Secretary of State for Aerial Navigation, having to build two sheds for very large airships, inaugurated a contest between French constructors. The Department of Military Engineering had charge of the contest and of the work, on behalf of the Air Service.

Each shed was to be 300 meters (984 feet) long, with sufficient space in the roof for five passage-ways one meter wide and at least two meters high. The front end, to be left entirely free, was to be outlined by a semicircle of 25 m (82 ft.) radius resting on a rectangle 25 x 50 m (82 x 164 ft.) (Fig. 3). The structure was to be able to withstand a wind of 150 kg per square meter (30.7 lb./sq.ft.) under the conditions imposed by the circular of 1903. These sheds were to be erected at the Orly airport, 17 km (10.6 mi.) south of Paris.

Under our supervision, Limousin and Company prepared and submitted an entirely original design, involving the use of a

* From "Le Genie Civil," September 22 and 29, 1923. For Part II, see National Advisory Committee for Aeronautics Technical Memorandum No. 333.

monolithic arch of reinforced concrete resting on the ground without other intermediary than a foundation slab. The cost of this proposal was found to be much the lowest to the Government, as regards both the original outlay and the upkeep, and the contract was accordingly awarded to Limousin and Company.

Since the work is being done by entirely new methods (Figs. 1 and 2), which give very satisfactory results, both in the perfect realization of the design and in the quality of the material, as also in the speed and economy of construction, we have decided to give, before the completion of the sheds, a detailed description of the means employed, in order to enable any who may be interested in these methods to obtain further information on the spot, while the work is in progress. We will preface this description by reviewing some of the steps which led us to the conception of the forms and methods of construction employed at Orly..

Historical sketch.- In 1913, we submitted to the Chief of Engineers of the District of Orleans proposals for airplane hangars made entirely of reinforced concrete, being probably the first to submit such proposals with a forfeit for non-performance.

One of these designs involved the use of a series of vaulted naves intersected by a like nave perpendicular to the preceding in such manner as to form a cross-vault. The description of the project was considered paradoxical, because it claimed that "For the construction of large airplane hangars

reinforced concrete will render it possible to realize conditions of permanence, safety, tightness and fireproofness hitherto unknown. In addition to all these advantages, there will be a saving in cost."

The war caused the abandonment of these projects, but they were revived in the winter of 1915-1916. Since the disadvantages of light hangars were becoming more and more manifest, we persuaded the Engineering Department to accept a type of large vaulted hangars derived from the 1913 plans.

A first series of eight hangars was constructed at Avord in 1916. Each of these hangars consists of a small circular vault with doors at each end. These vaults rest directly on the ground and are stiffened by ribs on the outside. The latter detail, though not appearing very important, has considerably affected the development of this type, because it has simplified the construction and greatly reduced the cost. In fact, by the construction of projecting ribs on the extrados, the forms are reduced to simple surfaces which can be quickly constructed by any workman. It also enables the removal of the forms in large sections without destroying them. The ribs are made after the vault, by using small auxiliary forms, which can be removed after 24 hours and can be used a great number of times.

The Avord hangars having been built very rapidly and being entirely satisfactory, the War Department entrusted to us the

construction of a new series of 31 hangars of the same type, to be erected at Istres (Bouches-du-Rhone), including three to be constructed as a single building, for use as a workshop. They differed from the Avord hangars only in length 42 m (137.8 ft.) instead of 60 (196.8 ft.) and in having only one door, the opposite end-wall being permanent (Fig. 4).

In the meantime (1916-1917), we submitted to the military authorities a series of designs of airports for airships, conceived on a very large scale. One of these projects included five sheds 92 m (301.8 ft.) wide and 42 m (137.8 ft.) high inside, radiating from an immense central court. These sheds were to consist of circular vaults strengthened by a double system of outside ribs. The development of the arch was about 132 m (433 ft.). Although the cost of reinforced concrete structures was then much less than at present, the estimated cost and time were still so great that the administration abandoned the project.

Lastly, we must call attention to an important application of the vaults made at Villacoublay in 1919 (Fig. 5). This was in connection with a hangar formed by three parallel barrel vaults 40 m (131.2 ft.) wide and 45 m (147.6 ft.) long, intersected by a fourth vault at right angles to the preceding and extending from the crown of the first to the crown of the third. The whole structure encloses a space 120 x 45 meters (393.7 x 147.6 ft.) without a column. This method of construc-

tion is characterized by the absence of all inside reinforcing ribs, which enables a very simple system of forms.

In addition to aircraft hangars, we have built a large number of factories, including several with very large naves. We had therefore had a very diversified experience with high edifices at the time the Orly program was undertaken.

General aspect of the problem.- All the difficulty lies in the magnitude of the absolute dimensions. It might be supposed that, after a type of structure had once been selected and methods of construction had been perfected by numerous and satisfactory applications to heights of 30 m (98.4 ft.), for example, it would be easy to adapt the type to heights twice as great by simply changing the scale. This idea, however, is entirely wrong.

If we should start with the design for an airship shed 30 m (98.4 ft.) high, built in the form of ribbed vaults, and double all the dimensions, we would obtain the following results: First, we would double the volume of concrete per square meter of the original design. The volume of the materials per unit volume of the structure would not change, but the laws of mechanical similitude indicate that the stresses would be multiplied by two, assuming everything else, especially the force of the wind, to remain the same. Now, it will be prudent to assume a stronger wind for a building 60 m (196.8 ft.) high than for one 30 m (98.4 ft.) high. Our struc-

ture would therefore be insufficiently stable, notwithstanding the greater thicknesses.

If we should try to improve the design without changing the general conception, we would have to increase greatly the height of the ribs in proportion to their thickness and to that of the vaults, which would necessitate the introduction of new members, in order to prevent the ribs from warping or buckling and to attach them more securely to the vault.

On the other hand, because of the very great absolute dimensions of these reinforcing members, we try to avoid them. We are then compelled to add a very complex framework to the vault, in order to give it the necessary rigidity, so that the structure loses the qualities of simplicity and the facility of construction which constituted the advantage of the original system.

Now, in the case of very high structures, these qualities (always desirable) become of capital importance, because the ratio between the cost of placing the materials and their cost, delivered on the job, increases very rapidly with the height at which they are used. This fact, well known by all builders, is due to many causes, first of all, disadvantages with regard to the personnel: time required and fatigue experienced by the men in reaching their working places, danger of falling, difficulties of supervision, greater effect of bad weather, more difficult recruiting of workmen and high wages;

in the second place, disadvantages connected with the material: higher and more exposed scaffolds and more powerful means for hoisting the materials. Consequently, a very high structure can be built economically, only when its method of construction is simple. The most promising projects from the theoretical viewpoint are without practical value, unless they satisfy this essential condition.

We have, therefore, come to the conclusion that large airship sheds are outside the limits of the economical application of the system of groined barrel vaults and that it is necessary to find a solution founded on a new principle.

Elimination of systems based on the use of precast parts.— At one time we contemplated the employment of parts cast in advance and assembled afterwards. This method, although very attractive in certain aspects and susceptible of some important applications, has, however, certain inherent and inevitable disadvantages, which are so great as to deprive it of all value in the case under consideration.

First, such structures require a covering composed of separate elements, such as tiles or slate. Now, these classic methods of covering lose much of their value when used at great heights, under conditions of exposure to the wind differing greatly from the conditions under which they are usually employed. At 50 m (164 ft.) above the bare plain of Orly, the force of the wind is diminished by no obstacle and its

action on the covering is very different from what it is on roofs of ordinary height. In storms, the rain and snow often get between the tiles, while the negative pressure on the ice side of the structure may generate a suction capable of tearing off the most securely attached tiles.

This would necessitate repairs, all the more difficult because they must be made with the greatest care and because they affect surfaces of considerable extent and difficulty of access. In order to make a fair comparison of such projects with monolithic structures, it is necessary to increase their original cost by the addition of a sufficient capital to provide for their upkeep.

Can this inferiority be offset by economy in materials or labor? The need for limiting the weight and bulk of the parts and for simplifying the assemblies leads to approaching more closely the forms and character of metal structures, rather than those of monolithic structures of reinforced concrete. One is led to employ very special elements, as in metal construction. There are arches, purlins, rafters, elements for covering, each performing its special role and not mutually aiding one another. Concrete belonging to a purlin or a covering element adds nothing to the general stability, which depends entirely on the arches, and inversely.

In a monolithic structure, on the contrary, the same concrete member, belonging to a vault, contributes effectively to the general stability against stresses of all kinds and in

all directions.

The first result of the specialization of the parts is their great number, which is a reason for economizing the material as much as possible. One is also urged in this direction by the need of reducing the weight of the parts to be handled and, in order to eliminate all material not strictly necessary, by constantly utilizing the limiting stresses and at the cost of researches regarding materials and forms. These researches are, moreover, facilitated by making the parts on a casting floor which can be prepared for obtaining the greatest efficiency of labor.

The assembling of these many parts produces complex structures of a slender appearance, similar to that of a metal structure, and which, just because of this aspect, give the impression of being economical, at least from the viewpoint of the quantity of materials used. This is, however, an optical illusion. A given volume divided into small scattered elements appears smaller than the same volume in a compact form. An open-work girder invariably appears lighter than a plain girder of the same total volume. A few well-placed openings allow the passage of sufficient light to produce on the eye a very remarkable impression of lightness, though, in fact, the volumetric reduction is insignificant and can be made up by a slight increase, inappreciable to the eye, in the exterior dimensions of the part. Logically, the advantage belongs

to the monolithic structure, in which the same portion of material can combine different structural functions. Concrete cubes are found to be generally of the same order, any advantage being easily due to the greater or less merit of one or the other research. From this viewpoint, the constant employment of maximum stresses and tested forms offsets the advantages of casting in place, which, however, has the advantage of lower mean stresses, thus affording the possibility of a saving in the weight of the cement and in the work of construction.

Monolithic construction is always far superior with regard to reinforcing. The same is true with regard to the developed areas of the forms, which are always much greater in a structure composed of precast parts. The relative inferiority of the latter method of construction tends, moreover, to increase with the absolute dimensions of the structures.

The comparison is no more favorable from the viewpoint of labor and equipment. The increase in the surface area of the forms offsets the ease of construction due to the casting on a prepared floor. This method is, moreover, costly and requires much space and valuable working equipment.

The principal difficulty, however, is the placing of the precast pieces. It is not possible to reduce their dimensions below certain limits and we find ourselves compelled to consider the lifting and precise adjustment of parts weighing up to 50 or more tons on scaffolds 50 m (164 ft.) high. Such operations require more powerful machines and stronger scaf-

folds, more money and more time than the simple placing of forms and reinforcements and the hoisting of an equivalent amount of concrete in buckets. The former method does not, therefore, even offer the advantage of more rapid construction.

In short, the system of construction with precast parts seemed to us to be inferior to the monolithic system. The course of events has demonstrated the correctness of this view. Structures to be erected with precast parts have indeed been proposed, but at prices averaging much higher than the cost of our monolithic structures.

Use of vaults with a double curvature.-- We then had the idea of developing a system which we had employed in 1915, for a structure designed to house a large glass foundry at Montlucon (Allier). This structure, 30 m (98.4 ft.) wide, consisted of a series of double arches, the extrados of which consisted of rectilinear elements connected by low vaults subtended by tie-beams, whose generatrices followed the slopes of the extrados of the arches (Fig. 6). This structure has stood remarkably well, despite the very severe conditions of contraction and expansion to which it is subjected by the furnaces.

Nevertheless, we abandoned the extrados formed by rectilinear elements and gave to the principal ribs the form of a catenary corresponding to the weight of the structure with a thickness increasing regularly from the crown to the imposts.

The secondary vaults, which connect them, accordingly present a double curvature, over in the direction of the main vault and the other at right angles to it, thus resembling an element of a torus.

It is well to note that, in this structure, the same as in a circular or barrel vault, there are no concrete elements which are inactive from the viewpoint of general stability, a prime point of superiority over the system employed at Mont-lucon. Two vertical planes parallel to the main ribs, and in any positions form, both in the secondary vaults and in the ribs, arches which under permanent loads, are stable almost to the buckling point. Consequently, the joining of the main ribs by tie-beams is not necessary for permanent loads alone, but only on account of the wind. The analysis of the stresses demonstrates, moreover, that the tie-beams can be less numerous and of smaller cross-section.

This conception, which constituted a great advance over our views previous to 1916, served as the basis of our original proposition for Orléans. The main ribs had been given a hollow rectangular cross-section, in order to save material (Fig. 7).

It was still susceptible of important improvements. It may be noted that a very large portion of the material is employed in the vicinity of the neutral axis and does not assist in the resistance to bending. Furthermore, the added bulk (i.e., the space between the intrados and the crown of the

secondary vaults) is large and uselessly increases the surfaces exposed to the wind.

From the viewpoint of construction, the presence of the tie-beams under the vaults, of ribs on the secondary vaults, and of hollow parts difficult to strip of the forms, constituted just so many complications. These disadvantages have been eliminated by a series of successive improvements.

Basic principles of the final project.- An airship shed must withstand only two principal types of stresses: those resulting from its weight and those due to the wind. Whatever may be the cross-sections of a monolithic structure of continuous arch, we will obtain, under the weight alone, a constant and minimum stress, by giving to the neutral axis of the arch the form of a catenary corresponding to the weight, with the single condition that the areas of the normal cross-sections of the arch increase from the crown to the supports, in accordance with a conveniently established law of variation. As to the effects of the wind, these depend entirely on the outside area, which it is desirable to reduce as much as possible.

The form which first occurs to one for obtaining the maximum moments of inertia, with the smallest bulk and the minimum material, is that of a hollow vault consisting of two concrete shells joined by ribs situated in planes perpendicular to the generatrices (Fig. 8). It is difficult to construct,

due to the great surface area of the forms required and to the fact that many of the forms lie inside of closed cavities, thus rendering their removal very difficult. This disadvantage is eliminated by the modification shown in Fig. 9. This also helps by reducing the exterior surface exposed to transverse winds, which are the most troublesome.

Another modification (Fig. 10) of this cross-section will render it possible to obtain, while keeping the same moments of inertia and using a very small additional amount of material, a very great diminution of the surfaces to be formed, together with great facility in removing the forms. This form is perfect from the viewpoint of simplicity of construction. A slight modification, to provide places for windows, gives the rib section of the sheds in course of construction (Fig. 11).

Principal structural elements.— We thus obtain a smooth-surfaced vault without ribs, extending from one support to the other, in the form of a catenary of its own weight. Rigidity is obtained simply by a regular corrugation of the surface, which produces a succession of forms resembling Zores irons (zee-bars or trough-plates) of very large dimensions, with neither girder nor projecting rib, either inside or outside.

Each corrugation has a total width of 7.5 m (24.6 ft.), making 40 corrugations for each shed. Fig. 12 is an exact representation of the crown section of one of these elements, with the dimensions. All the other normal sections are

derived from this one by a simple oblique projection on planes passing through its base line. The obliquity of the projection increases from the crown to the impost, in such manner as to obtain the law of variation of the heights and thicknesses which is represented by the graph (Fig. 13). This curve was determined by approximations, in such a way as to obtain the maximum stresses of the same order throughout the whole extent of the structure. The total height of the cross-section varies from 3 to 5.4 m (9.8 to 17.7 ft.) and the moment of inertia of each element varies from 0.76 m^4 (2.5 ft.⁴) to 4.3 m^4 (14.1 ft.⁴).

The walls rest directly on foundations consisting of continuous slabs of reinforced concrete one meter thick, which, in turn, at a depth of two meters below the floor of the shed, rest on sandy clay of medium consistency, quite uniform and too thick to be penetrated for the sake of finding a more solid foundation.

The pressure on the ground was limited to 1.5 kg/m^2 (.3 lb./ft.²), including stresses produced by a wind pressure of 150 kg/m^2 (30.7 lb./ft.²). The horizontal component of the stresses, at the point of contact of the concrete with the ground, makes a very small angle with the horizontal, the vertical load (weight of structure, of the foundation and retaining walls) being about 72,000 kg (158,732 lb.) per running meter and the horizontal thrust only 10,400 kg (22,928 lb.).

Conditions of stability of the structure described above.

I. Effect of permanent load (weight of structure).- Disregarding the effects of elastic contraction, the condition realized by the structure, namely, that of having its neutral axis coincide with the catenary of its own weight, results in all the normal sections being uniformly compressed, the catenary diagram giving the value of the resultants at each point of the neutral axis. The stresses R_b are equal to N/ω , N being the resultant and ω the area of the cross-section. These stresses must be increased by those due to linear variations (contraction and expansion) to which we may add the effect of elastic contraction (or shortening).

However, in an arch, the stresses due to linear variations decrease very rapidly, when the ratio of the camber (or rise) to the mean height is increased. This ratio is here quite large and such that the effect of the linear variations, including the elastic contraction, is very small and may be neglected without disadvantage.

Made as indicated above, the computation of the stresses due to the permanent loads alone, gives the following results:

$$\text{Crown } R_b = 6.7 \text{ kg (14.8 lb.)}$$

$$\text{Imposts } R_b = 9 \text{ kg (19.8 lb.)}$$

II. Effect of the wind.— The wind produces complex effects. If we consider the general resultants of the wind, applied between two sections made at the points S and $S + dS$ of the central axis, acting on the arc constituted by one of the Zores or zee elements, we obtain, by the usual methods of computation of elastic arches, the normal and tangential stresses and the moments of flexure. These stresses were calculated according to the following hypotheses:

1. The hypothesis imposed by the conditions of the competition and by the contract: a wind pressure of 150 kg/m^2 (30.7 lb./ft.^2) acting under the conditions indicated by the circular of 1903 (direction 10° to the horizontal, rule of $\sin \frac{\theta}{2}$ of the angle of incidence);

2. Following the hurricane of March 7-8, 1922, during which pressures were found very much in excess of 150 kg/m^2 , the "Service de la Navigation Aerienne" asked us to determine whether the stability continued to be satisfactory for a wind pressure of 250 kg/m^2 (51.2 lb./ft.^2);

The improvements introduced into our project between the filing of the bids and the approval of the plans of execution enabled us to substitute this figure of 250 kg for the former 150 kg , without additional expense;

3. The calculations were repeated, with allowance for an internal pressure capable of reaching 150 kg/m^2 (30.7 lb./ft.^2) in the case of a wind of 250 kg/m^2 (51.2 lb./ft.^2);

4. We substituted, in the somewhat arbitrary distribution resulting from the rule of the \sin^2 , various distribution laws derived from experimentation with small models.

The magnitude of the bending moments under the most unfavorable hypotheses is represented in Fig. 14, by plotting, at right angles to the neutral axis, the value of the corresponding bending moment for each section. The wind produces large bending moments, presenting maxima at the imposts and two other less important maxima at intermediate points on the neutral axis.

The shearing stresses accompanying the moments are relatively very unimportant and it is useless to take account of them.

The ordinary rules for computing compressed and bent parts render it possible to deduce from the values obtained for the bending moments, allowing for the effects of the permanent loads already known, the stress figures given below. (We have neglected the effects of snow, since its accumulation is not possible in a strong wind, due to the shape of the structure. They would amount, moreover, to only a fraction of a kilogram.)

Stresses due to permanent loads and to a wind pressure of 150 kg/m² (30.7 lb./ft.²) in the sections where the bending moment is the greatest (allowing for the negative pressures at the points where they increase the stresses.)

Crown

Highest point of section, $R_a = 0.8 \text{ kg}$ (1.76 lb.)
 $R_b = 26 \text{ "}$ (5.73 ")

Lowest point of section $R_a = 2.2 \text{ "}$ (4.85 ")
 $R_b = 19 \text{ "}$ (4.19 ")

Imposts

Highest point of section, $R_a = 0.6 \text{ kg}$ (1.32 lb.)
 $R_b = 26 \text{ "}$ (5.73 ")

Lowest point of section, $R_a = 0.2 \text{ "}$ (0.44 lb.)
 $R_b = 28 \text{ "}$ (6.17 ")

It is apparent that, up to a wind pressure of 150 kg/m^2 (30.7 lb./ft.^2), the reinforcing, so to speak, does not work and that the maximum stress of the concrete reaches only 28 kg/cm^2 (398 lb./ft.^2).

For a wind pressure of 250 kg/m^2 (51.2 lb./ft.^2), the stress figures become:

Crown

Highest point of section, $R_a = 6.5 \text{ kg}$ (14.3 lb.),
 $R_b = 50 \text{ "}$ (110 lb.)

Lowest point of section, $R_a = 13.5 \text{ "}$ (29.8 ")
 $R_b = 38 \text{ "}$ (83.8 ")

Imposts

Highest point of section, $R_a = 14.2 \text{ kg}$ (31.3 lb.)
 $R_b = 58 \text{ "}$ (127.9 lb.)

Lowest point of section, $R_a = 12 \text{ "}$ (26.5 lb.)
 $R_b = 56 \text{ "}$ (123.5 lb.)

These figures were calculated on the assumption that the concrete would not take tension. On the contrary assumption, they become:

Crown

Highest point of section, $R_a = 1.0 \text{ kg (2.2 lb.)}$
 $R_b = 34.0 \text{ " (75.0 ")}$

Lowest point of section, $R_a = 1.4 \text{ " (3.1 ")}$
 $R_b = 30.0 \text{ " (66.1 ")}$

Imposts

Highest point of section, $R_a = 1.6 \text{ kg (3.5 lb.)}$
 $R_b = 35.8 \text{ " (78.9 ")}$

Lowest point of section, $R_a = 1.6 \text{ " (3.5 ")}$
 $R_b = 36.0 \text{ " (79.4 ")}$

The real values of the stresses certainly lie between those extreme figures, the highest of which remain within the limits established by the circular of 1906, our steels being drawn and possessing an elastic limit of over 30 kg (66.1 lb.) and the concretes employed possessing a resistance to compression of at least 200 kg/cm² (2845 lb./ft.²) after 90 days.

The stresses thus determined are not the only ones caused by the wind. In fact, the above calculations assume, as an essential condition, the indeformability of the transverse section of the element considered as an arch. Now, two distinct categories of forces tend to produce deformation.

In the first place, the local pressures due to the wind, which act at right angles to the surfaces and which are approx-

imately equal to the pressures exerted by the wind on a right cylinder enveloping the structure and which can be evaluated by the customary rule of the \sin^2 . The whole of the solid ABCD (Fig. 15) functioning as an arch, balances these forces by reactions evidently disposed along BA and CD.

In the second place, it is well to make allowance for the secondary stresses or loads, accompanying these flexures. If we consider a right section, the bending moments due to the wind introduce compressions and tensions, as represented by Fig. 16.

If we now examine a section AB, we find that it exerts simple compressions along AA' and equal tensions along BB'. Between AA' and BB' there is a gradual transition from tension to compression. The compressions along AA' produce pressures toward the exterior of the structure. The tensions along BB' produce pressures toward the interior. These pressures are equal to the quotient of the compressions or tensions divided by the radius of the curve of the lines of application of these loads. The whole of these forces constitute a system in equilibrium.

Lastly, we find ourselves in the presence of two sets of forces in equilibrium, tending to deform the right section. Calculation immediately shows that the latter can resist only at the expense of an enormous increase in the thicknesses and in the reinforcing. We would, in fact, find, for the thick-

nesses and reinforcing adopted (in the actual structure) in a section located near the crown (one of the sections the most stressed by the loads we have been discussing), that the bending moment at the point C, for a one-meter section (tranche), is 2300 kg (5071 lb.), which gives:

$$R_a = 134 \text{ kg/mm}^2 (190595 \text{ lb./ft.}^2);$$

$$R_b = 360 \text{ kg/cm}^2 (5120 \text{ lb./ft.}^2).$$

In order to reduce these stresses to acceptable magnitudes, it would be necessary to multiply the sections of concrete and steel by about 4, which would cause the structure to lose its economical character. It is therefore absolutely essential to prevent the deformation of the sections. The best way consists in fitting tie-beams between the points C and F, and G and L (Fig. 17).

Each Zores or zec element is thus transformed into an arch of special form. We thus find a close relationship between the forms of actual construction and those of the original project, comprising a series of secondary vaults joined at the ribs.

We stated that the tie-beams could be placed either inside or outside. The inner position offers some theoretical advantages. Nevertheless, we have discarded it for structural reasons which seemed conclusive. In the first place, the position of the tie-beam determines the form of the unit of construction, since each unit must have inherent stability at the

moment of removing the forms. The lower tie-beam supports elements like ABCD (Fig. 18), the top of the shed being at A.

The successive elements are therefore joined at the gutter, a portion which is preferably made in one piece for the sake of tightness. The cross-section is much more easily divided between the successive units of construction in the portions BC and B'C', where it is thin and interrupted by window openings throughout most of its length, than at DA', where it is very thick.

Moreover, the pressure of the liquid concrete is greatest at D', where it follows the line of the temporary form, which must be fitted between the closely spaced reinforcing rods at the edge of the last element to be poured. This would be a very difficult piece of work.

In the second place, the men would have to work constantly on the slope C'D' which would be very much exposed to the wind and very dangerous. Lastly, and especially, the tie-beams, such as AD and A'D', would pass through the inner form and deprive it of an essential advantage, namely, the ability to be removed in a single piece without hindrance. Although this point is of secondary importance, let us note, in passing, that the upper position of the tie-beams presents some advantages from the viewpoint of expansion. In particular, the structure retains complete liberty of expansion in all its upper portion, the tie-beams being, in this region, completely

interrupted by the skylight, which forms a flexible joint. This might be important in other applications.

On the contrary, the upper position of the tie-beams entails rather serious difficulties for the good attachment of the end walls. The end of the shed is represented by Fig. 19. The importance of the end wall is increased and it is difficult to secure it. This difficulty has been overcome by adopting, for the two semi-elements next the ends, a closed hollow cross-section stronger than the usual cross-section and capable, due to its large tubular form, of withstanding very great torsional moments, which enable it to transmit to C, from the points A and B, the pressures exerted by the wind.

This reinforcement has many other advantages, especially that of facilitating the removal of the soffit scaffolding of the top element which, after this operation and until the scaffolding is installed again for the next element, remains entirely unsupported and exposed to the wind.

According to the diagram of the secondary stresses in the transverse section of the vault, with the tie-beam outside, the resulting stresses R_b represent hardly the tenth part of the stresses found in the absence of the tie-beams. It is remarkable to find that these capital pieces are subjected to relatively insignificant stresses, since their elimination would entail complete instability of the structure under the action of a medium wind.

Accessory Elements

Tie-beams.-- We designate as tie-beams the light elements on the extrados whose absolute necessity for the stability of the structure we have just shown, although they may act as either tension or compression members, according to the case. They are square members 0.14×0.14 m (5.5 x 5.5 in.) reinforced by six 10 mm (.4 in.) rods located about 10 m (32.8 ft.) apart. They connect the two sides A and D (Fig. 19) of the same element. The stability conditions do not require their continuity from one element to another. One of these tie-beams, placed in the axis and at the top of the shed, was provided with a hollow space 0.8 m (31.5 in.) wide, which can be utilized as a passageway from one end to the other of the shed.

End walls.-- The shed is to be closed at each end by two large doors of structural steel. They are to be independent of the rest of the structure and stable in themselves. They will be built under a separate contract. The movable doors have an area smaller than the total cross-section of the shed. It would, in fact, be useless to include in the movable parts of the end walls, the portion of the shed occupied by the inspection passageways and stairs. The area of the doors was therefore limited to a parabola with a vertical axis 50 m (164 ft.) high and intersecting a 70 m (229.7 ft.) base on the ground (Fig. 20). This area is divided along its central line,

into two sections, one opening to the right and one to the left.

The space between the doors and the inner side of the vault is closed by an arch of windows 5.37 m (17.6 ft.) wide at the base, 1.8 m (5.9 ft.) at the haunch and 3.8 m (12.5 ft.) at the crown. As already indicated, the gable end consists of a barrel vault having the same thicknesses as the reinforced portion of the intrados of the running elements and forming an element of a large tube of triangular cross-section (Fig. 19).

The frame of each gable consists essentially of a rib in the form of a gutter designed to collect the water running along the top of the shed and to empty it outside the doors. This rib constitutes an arch almost able to support itself without buckling.

In order to prevent buckling and withstand the force of the wind, this rib is supported, at intervals of about 9 m (29.5 ft.) by members practically perpendicular to the contour of the opening and lying in its plane, each such member forming a rigid triangle with an oblique member resting against the intrados.

Between the rib and the intrados the space is filled up to a height of 16 m (52.5 ft.) from the ground, by a wall of reinforced concrete; above this, by continuous glass windows, whose muntins are perpendicular to the contour of the arch (Fig. 20).

Small doors.- In addition to the principal doors for the entrance and exit of airships, each shed has ten side doors, four of them being 1×2 m (3.28 x 6.56 ft.) and six 4×4 m (13.1 x 13.1 ft.). The latter, which exceed the width of the narrow element of the extrados designed to receive the windows, necessitated a modification of the normal cross-section, in place of which there was substituted another cross-section having two parallel elements separated by the width of the door, provided with rabbets for the latter and affording at least an equivalent moment of inertia. The two sections are joined by reinforced concrete girders capable of transmitting all loads from one section to the other.

In designing these elements, care has been taken to avoid any projection with reference to the inner surface of an ordinary element, in order to enable the utilization of the same form for elements provided with doors as for the ones without doors.

Lighting.- A portion of the cross-section of reduced thickness is reserved for the window frames. There are 62 of these for each element or 2480 for the whole shed, with a glass area of 2781 square meters. They are cast on the ground, using a rich mixture, and sealed into the vaults as soon as the latter are made. They are provided with special reinforced glass of a yellow color, in order to protect the fabric of the balloonets from harmful solar radiations. This motive (not to

speak of the original cost and the upkeep of glass areas) renders it advisable, in airship sheds, to avoid excessive areas of glass. On the other hand, the difficult work of airship building requires good illumination. These conflicting viewpoints seem to have been quite happily harmonized at Orly, by the good distribution of the windows, as has been confirmed in the portions of the sheds already completed.

The ventilation required special attention. It must be abundant in order to render impossible the accumulation of explosive gases in the top of the shed. On the other hand, the protection against rain must be as perfect as possible.

In order to satisfy these conflicting requirements, we devised the following plan. On top of the shed, the zone reserved for windows is left entirely open on an arc 30.m (65.6 ft.) long. Above this vacant area, the glass surface is raised and supported on elements made up of superposed sections like Persian blinds (Figs. 21-23). Water-charged air entering the orifice D is slowed down in the chamber A and the drops of water are stopped by the wall B. The air then passes through C at a lower velocity than it had when it entered D and which is now insufficient to carry along the water running down the wall B.

Walkways, tracks and stairs.-- Five longitudinal walkways affording a free passage 1 x 2 m (3.28 x 6.56 ft.) were stipulated in the plans. They are provided with rails enabling the

easy transportation of suspended loads. The specified loads were at first: for the central rail, a uniform load of one metric ton (1000 kg - 2204 lb.); for the lateral rails, a single load of two tons. They were subsequently raised to a single load of four tons for the lateral walkways and to a single load of ten tons for the central walkway.

These walkways (Fig. 24) are made of reinforced concrete. They are cast on the ground and then suspended from the arches, after the concrete has hardened, by tie-beams of reinforced concrete, at intervals of 7.5 m (24.6 ft.) (or one for each element). Means of access to the walkways (stairs and lateral passages) are provided by similar methods.

Construction

Importance of materials and methods of construction.— For clearness of exposition, it was necessary to describe the structures before considering the materials and methods of construction. Although this order (first the design and then its execution) may seem very logical, such, however, is not the case. For all structures, however unpretentious, the preparation of the design presupposes a knowledge of the materials and methods of construction.

It is not necessary for a study of the materials and the manner of employing them to precede and govern the study of the structural forms in the case of ordinary structures erected by well-known methods. When, however, it is proposed to carry

out a new project with new materials and methods, the study of these means must keep pace with the designing of the structure. The most important advantage of public competitions is the facilitating of the simultaneous study of the designs and of the means for executing them.

The novelty of the Orly project made the study of the means for carrying it out our first concern. In fact, the forms described were specially designed and adapted to the materials and methods of construction selected, which constituted the principal object of our researches. These researches covered the mixing of the concrete, the casting, the supporting and care of the forms, the conveying of the materials and the reinforcing.

Composition, mixing and casting of the concrete.- These points are very important, since the method of construction, about to be described, depends entirely on certain properties of the concrete. It is too often believed that the data given in the circular of 1906 regarding certain types of concrete made with 400 liters (14.125 cu.ft.) of sand and 800 liters (28.25 cu.ft.) of gravel mean that these proportions are the best in all cases and must necessarily be included in the specifications. As a matter of fact, the committee on reinforced concrete simply investigated certain mixtures and reported the results obtained. It chose the ratio of 400 : 800, because Paris contractors frequently employed a mixture of

sand dredged from the Seine and 800 liters of "mignonnette" gravel of the same origin, for no other reason than the convenience of the ratio of 1 : 2. Now, there is no error more serious than that of adopting uniform proportions without regard to the nature of the materials used or the end in view. The proportions of sand, gravel, cement and water can be determined only by testing, on the job, the materials actually to be used and taking account of the conditions of use.

It must be remembered that the strength of the test-cubes in the laboratory is not the only quality of a concrete. Moreover, it is often only distantly related to the real strength of the concrete when placed in the structure. This is especially true of dry concretes, formerly so much recommended.

The recent great progress in the manufacture of cements renders it easy to obtain, with good available materials, a strength, after 90 days, ranging from 150 (330.7) to 400 kg (881.8 lb.) and even higher.

In the great majority of cases, the attempt to obtain strengths above 200 kg (440.9 lb.) or even 150 kg (330.7 lb.) is utterly useless. It is better to obtain the qualities of plasticity, fluidity and homogeneity, enabling a good encasing of the reinforcing rods and a pleasing appearance of the surfaces. This viewpoint nearly always leads to the adoption of concretes richer in sand than the 400 : 800 ratio, even coarse, gravelly sands without the addition of any gravel,

and the use of more water.

This is the solution to which we were led by our experiments at Orly. In order that the system of construction which we intended to use should give the best results, it was necessary to obtain concrete having:

1. Density sufficient to give smooth surfaces and perfect impermeability, even without the use of paint;
2. Fluidity sufficient to completely fill the forms, which could not be tamped and into which the concrete could be poured at only a few points;
3. Rapid hardening, enabling the quick removal of the forms.

We found that all the necessary qualities were possessed by a mixture of $1000/(35.3 \text{ cu.ft.})$ of gravel from the gravel banks of Villeneuve-le-Roi, situated 4 km from the airport, with 350 kg (771.6 lb.) of good Portland cement, prepared with a rotary gravity mixer and poured almost liquid into the forms, which were simply jarred by repeated blows.

When passed through a wire screen with 5 mm (0.2 in.) meshes, this gravel was found to contain 56% of sand and 44% of gravel proper. The mixture differed greatly, therefore, from the ratio of 400 : 800. The quantity of water required to produce the desired plasticity is very slightly (1 - 2%) in excess of that which gives plastic consistency. It is not necessary to measure the water. Measuring could be of real

use only if the degree of humidity of the gravel were known. This could be determined, however, only at the price of inadmissible complications. Moreover, for experienced workmen, the appearance of the concrete is a sufficiently accurate indication.

The forms are jarred by means of pneumatic hammers or vibrators. By this method a rather stiff concrete acquires properties similar to those of a perfectly liquid concrete. The free surfaces become horizontal; the concrete adheres perfectly to the reinforcing; the surfaces are smooth and the edges well-defined. Moreover, the concrete becomes very dense, so that it is perfectly impermeable and sets considerably quicker.

On the other hand, this method produces considerable pressures in the forms, necessitating particular care, especially as we contemplated rapid construction, i.e., the placing of a very great height of liquid concrete in the form before it began to set. This great rapidity of pouring is an important condition for the successful execution of the work, the piezometric (compressive) pressure thus obtained being a factor which tends to increase the density of the concrete and insure the perfect filling of the forms.

We found that an excess of water, even considerable, was no disadvantage. As a result of the jarring, any excess water filtered through the forms or came to the top without carrying away any of the cement or rendering the mixture less

dense. The surface water can, at the close of work, be absorbed by drier concrete. An insufficiently moist concrete, on the other hand, gives poor results, the concrete being less dense and the surface rougher. There may even be cavities between the reinforcing and the forms, which have to be "patch-filled."

The strength found for 30 mm (.787 in.) cubes, made with Demarle and Lonquety cements and poured under the same conditions as the building, were as follows, the tests being made in fair weather.

After 2 days, 31 kg (68.5 lb.)

"	3	"	47	"	(103.6 ")
"	7	"	85	"	(187.4 ")
"	15	"	121	"	(266.8 ")
"	28	"	144	"	(317.5 ")
"	90	"	300	"	(440.9 ")

In winter, at very low temperatures, the setting progressed a little slower.

The maximum stresses in the region of the crown being only 6.7 kg (14.8 lb.), without taking account of the wind, it is obvious that, after the third day, there is a large safety factor with reference to the permanent loads. As regards the effect of the wind, it should be noted that the last section poured is supported, on the one hand, by the sections previously poured and, on the other hand, by the centering, which

has been put in its new position and contributes to the rigidity of the vault, after a relatively short period for the transfer. In winter, four or five days must be allowed for sufficient setting. Test-bars, reinforced on only one side, are used for estimating the degree of setting (Fig. 25). These are tested in bending between supporting points 0.6 m (23.6 in.) apart, by means of a machine improvised on the job (Fig. 26).

The centering may be removed, when the load required to produce rupture at the middle of the test-bar reaches 20 kg (44.1 lb.), which corresponds to a bending moment of 30 mkg (317 ft.-lb.), or a stress R_b of

$$44 \text{ kg (97.0 lb.)}, \text{ if we assume } m = 10,$$

$$38 \text{ " (83.8 ")} \quad " \quad " \quad m = 20,$$

$$30 \text{ " (66.1 ")} \quad " \quad " \quad m = 30.$$

There is evidently some uncertainty regarding the value of m , on account of the incomplete setting of the concrete, whose modulus of elasticity is still quite different from the ordinarily accepted values, but this uncertainty can be removed by comparisons with direct crushing tests. The test-pieces are made from the last batch of concrete for filling each form, jarred in the same manner and left to set on the centering beside the last concrete poured. Since the small volume of the test-pieces is unfavorable, both with respect to their casting and the setting of the concrete, it is certain

that the strengths of the concrete indicated by the tests are not quite as great as those actually obtained in the structure.

Construction of the forms.- In order to obtain the best results, the forms must be perfectly tight, smooth on the inside, not adhering to the concrete, very strong and very resistant to deformation. We use wooden forms for the following reasons: They are the lightest for the same strength, the most economical and the easiest to repair or adjust in case of accident or of any slight error in design. Well-oiled wood does not adhere much to concrete, even in freezing weather and, in any case, much less than metal. It gives no trouble from temperature/^{expansion} Furthermore, the important advantages of wooden forms are as follows:

1. They can be made very rapidly with the ordinary resources of a reinforced-concrete plant, both as regards materials and personnel. Therefore, delays in construction depend only on ourselves, since supplies of lumber of suitable dimensions can always be quickly obtained, even in large quantities. For steel forms, we would have had to await the delivery of the sheets and necessary fittings and then be subjected to delays in construction, over which we would have had no direct control.

2. In winter, double walls of oiled wood provide the concrete with an excellent protection against the cold. During the most severe cold spells (down to about -12°C), the use of

warm water in mixing the concrete was all that was necessary to insure its setting under absolutely normal conditions. The wooden forms retained the heat of the mixing-water and that from setting and maintained a temperature sufficient to enable setting and the beginning of hardening.

The resistance of wooden forms to wear proved entirely satisfactory. After being used 40 and even 80 times, they were practically as good as new. On the basis of a large number of observations of deformation, we fixed upon 6000 kg/m^2 , (1229 lb./ft.^2), as the maximum inside pressure the forms should be able to withstand without appreciable deformation. In order to obtain this stiffness, we made the forms (Fig. 27) of good spruce planks planed on all sides of a uniform thickness of 35 mm (1.38 in.) and securely nailed to frames placed at intervals of about 0.8 m (31.5 in.). These frames were formed by two very rigid pieces placed edgewise and separated by a space of 5 cm (1.97 in.). The two walls of the form were held together at regular intervals by 22 mm (.87 in.) bolts, which passed through the wall and both sides of the form between the edgewise pieces and bore on the latter through supporting plates of sheet-steel, so as to prevent their spreading.

For the double purpose of facilitating the removal of the bolts and of enabling an initial setting up of the forms sufficient to prevent any appreciable deformation during the filling, the two sides of the form were separated by blocks of concrete of the same composition as the wall to be poured,

each block being of octagonal shape with a cylindrical hole for the passage of the bolt and a thickness exactly equal to the thickness of the wall. The bolts were tightened until the concrete blocks indented the sides of the form. Under these conditions, the largest charges of concrete never caused the least spreading of the forms and the desired thicknesses were obtained to within a millimeter, a result rarely obtained in the plant. This is very important, both from the viewpoint of cost (due to the large area over which any excess thickness would extend) and from that of the agreement of the actual weight of the structure with the assumptions made in the design.

The thicknesses being thus accurately fixed, one of the form walls can consist simply of panels of relatively small dimensions (about 1.6 m = 5.25 ft. high) with no other stiffening members except the timbers on edge at its junction with the first wall. The joints are located at the middle of the 5 cm (1.97 in.) space separating the two edgewise pieces, it being then only necessary to define the form of the second wall. This is done in assembling, by simply nailing to the edgewise pieces, forming the frame of the wall, other pieces of like dimensions, forming with the first a complete triangular structure which rests on the support of the forms. The junction between the latter and the parts previously poured is effected very easily by the simple clamping action of the

forms. On the opposite side, the ends of the reinforcing rods are carried through two notched pieces of wood against which both walls of the form are clamped.

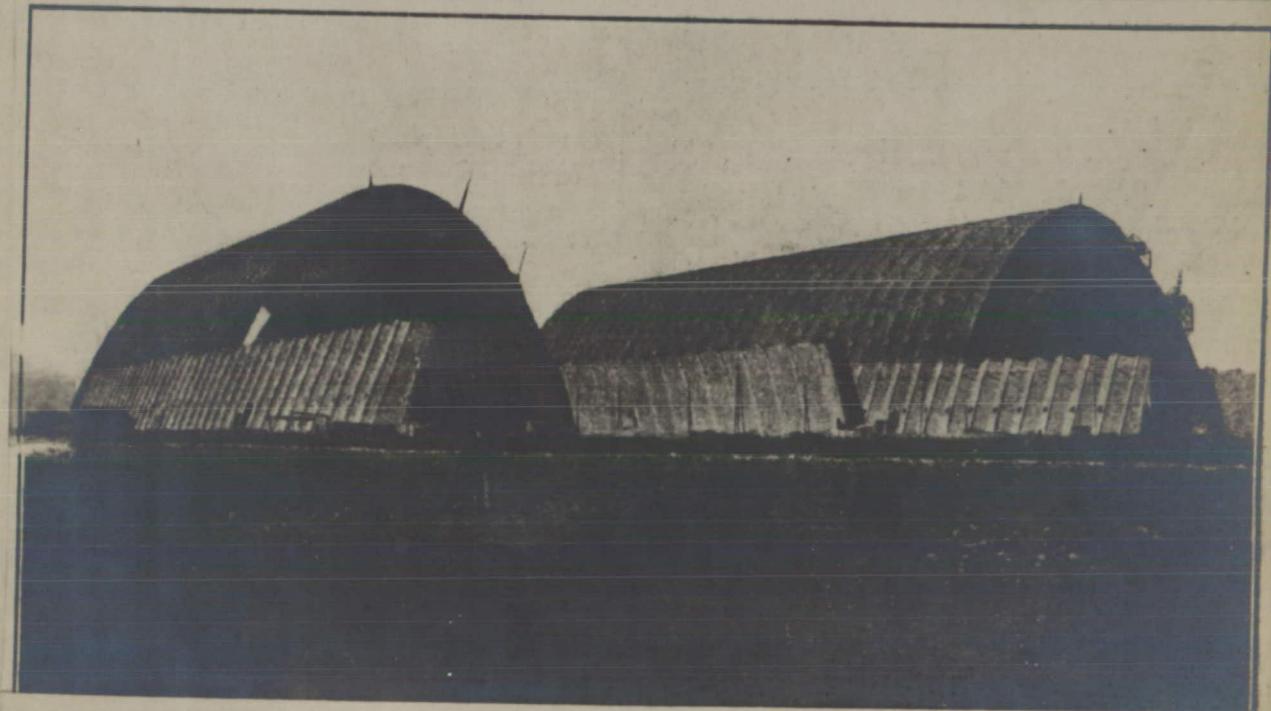
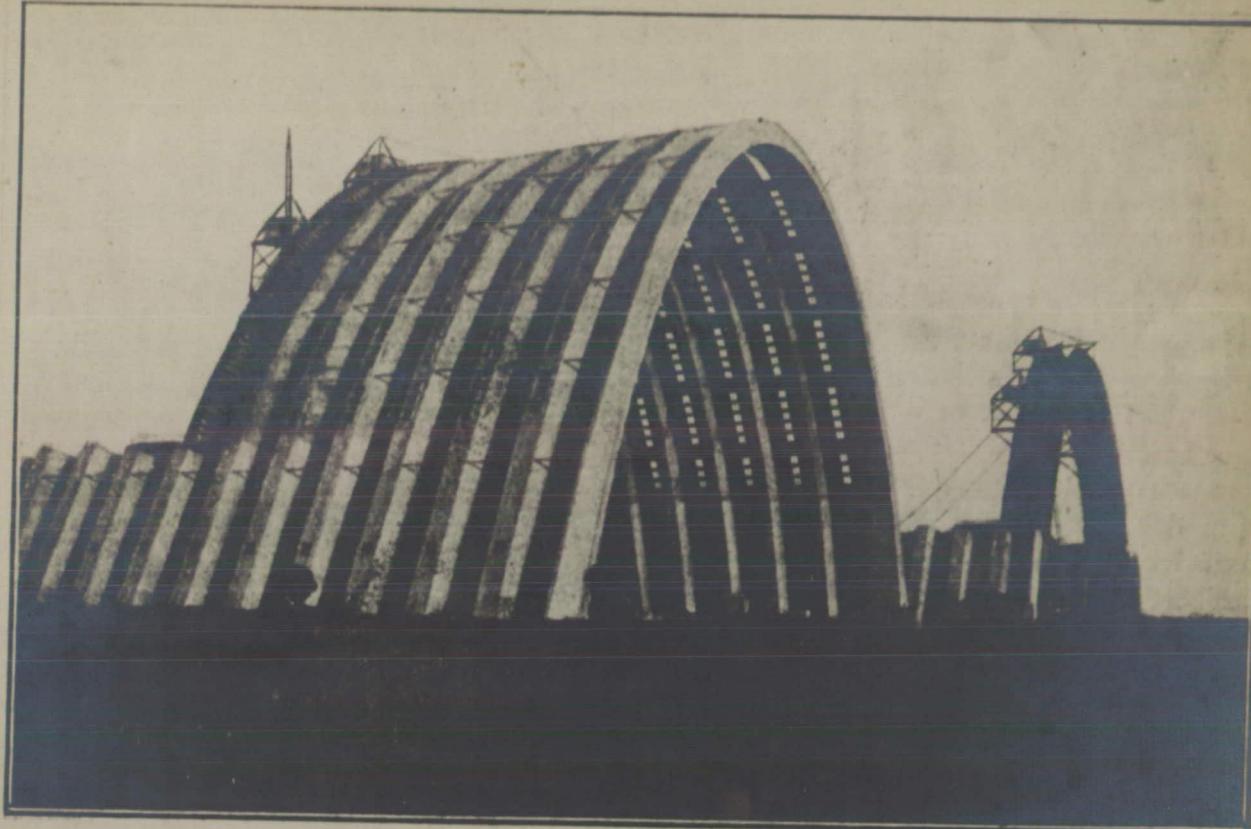
Dimensions of the forms.— Since each shed consists of 40 identical elements, excepting the top elements, the whole of both sheds can theoretically be poured with the use of a single form 7.5 m (24.6 ft.) long, included between two vertical planes passing through the middle of two successive skylights.

Considering the time required for designing and making the form and transferring it from one shed to the other, together with the time limit imposed (two years), it would have been necessary to reduce the time consumed in making one 7.5 m section to about three days. This result could have been attained by employing quick-setting cement and machines, for handling, a little more powerful than the ones we actually used but we thought it better to make special forms for each shed and continue to use ordinary cements. The forms for the bottom sections were, however, used for both sheds and were accordingly employed 80 times.

By employing a 7.5 m (24.6 ft.) form for each shed and allowing an average of nine days for the construction of each element, including the placing of the forms, pouring and setting (requiring from three to five days according to the season, with good ordinary Portland cement) and removing the forms, there still remained about a year for working out the details

of the forms, their construction (which could be done while the foundations were being laid) and the construction of the windows, walkways, etc., which work could keep pace with the construction of the shed. All the special forms (for doors, ribs and walls) were made separately for each element.

Translation by Dwight H. Miner,
National Advisory Committee
for Aeronautics.



Figs.1&2 Airship sheds at the airport of Orly
(Views taken in April and September, 1923)

enlarge to 10"

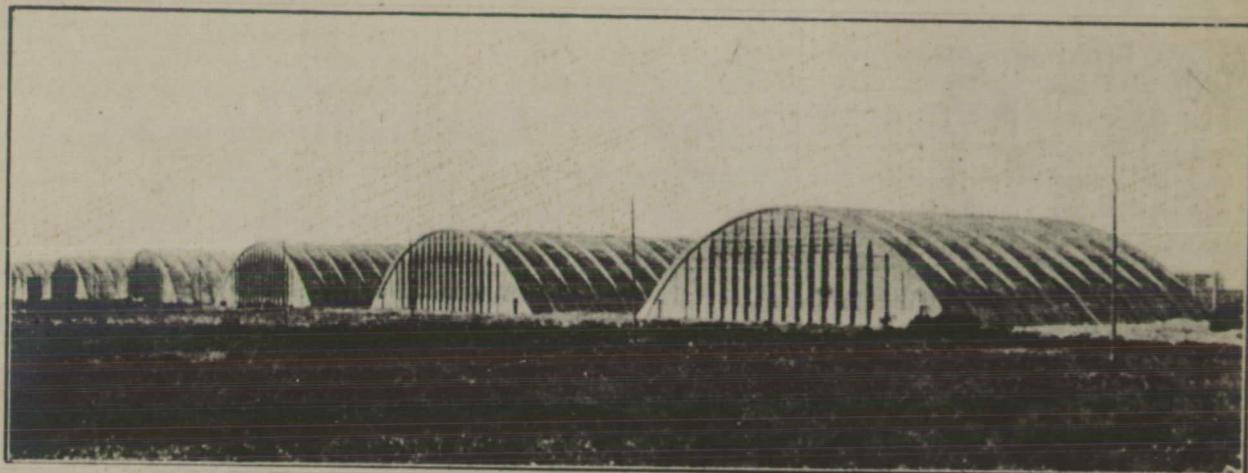


Fig.4 Istres airplane hangars of reinforced concrete

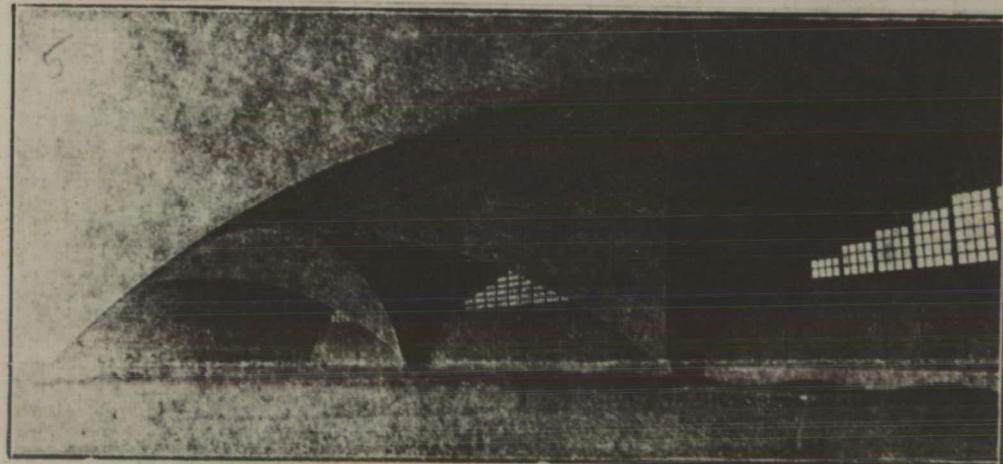


Fig.5 Interior view of Vallacoublay hangars,
showing groined vaults.



Fig.6 Vaulted roof,
of reinforced
concrete, of
glass foundry
at Montlucon.

Photostat - Enlarge 20 10"

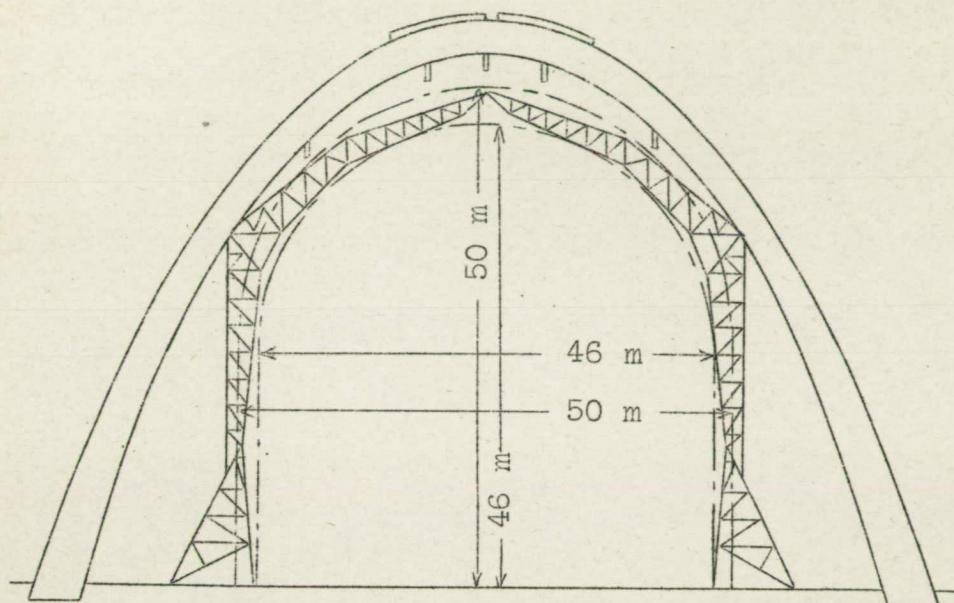


Fig.3 Comparison of the Orly shed with the one at Scott Field, U.S.A.

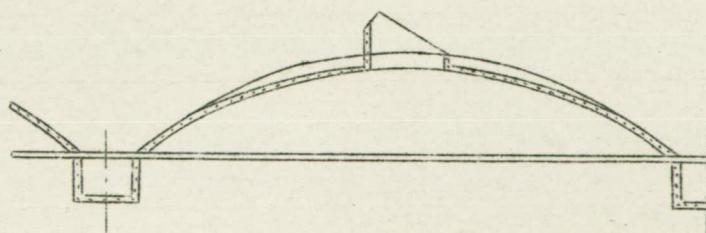


Fig.7 Partial section of a reinforced-concrete vault.
(Original project of Orly airship sheds.)

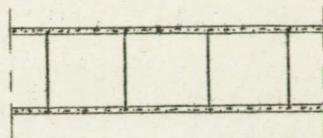


Fig.8

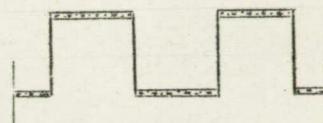


Fig.9

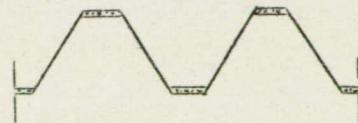


Fig.10

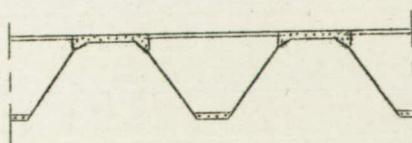


Fig.11

Diagrams of cross-sections of vaults of reinforced concrete
for an airship shed.

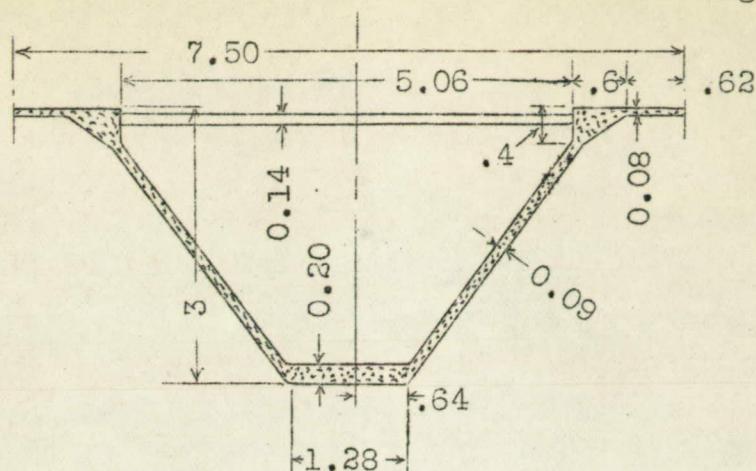


Fig.12 Cross-section, at the crown, of an element of the vault of the Orly sheds.

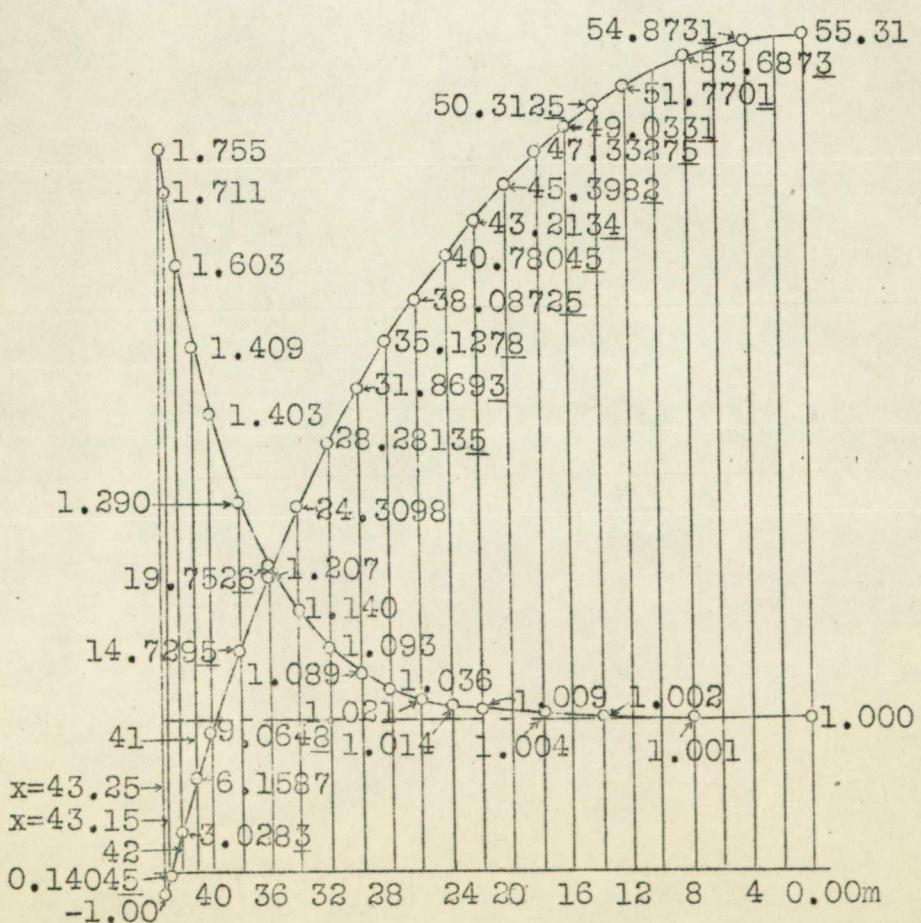


Fig.13 Diagram of the neutral axis of an arch and the law of thickness.

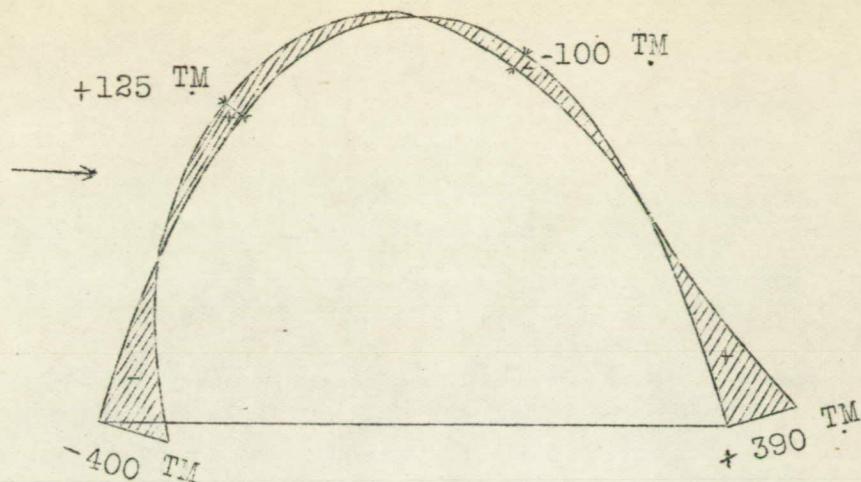


Fig.14 Diagram of the bending moments due to the action of the wind on the length of a half-rib of the vault 3.75 m (12.3 ft.) Wind pressure = 250 kg/m^2 (51.2 lb./ft.^2) Neg. inside pressure = 150 kg/m^2 (30.7 lb./ft.^2)

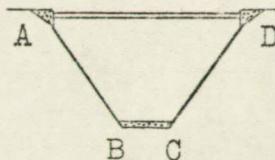


Fig.15

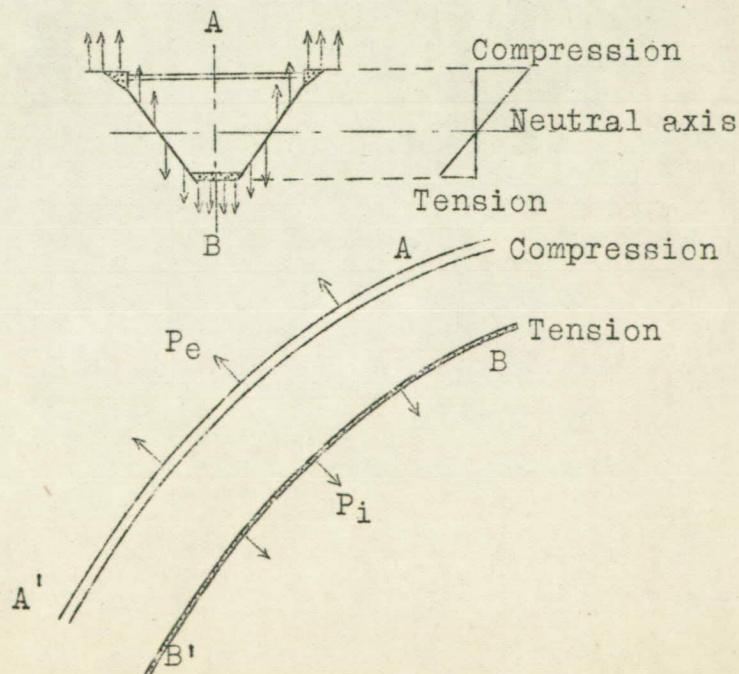


Fig.16

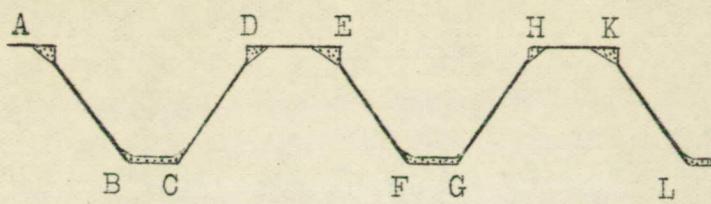


Fig.17

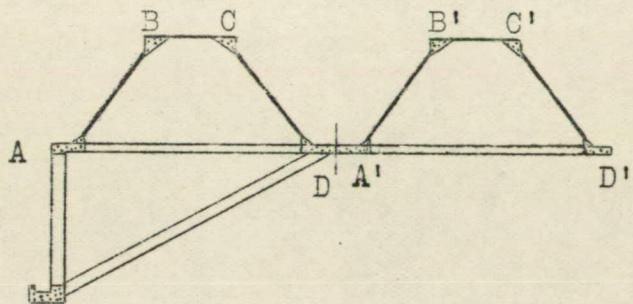


Fig.18

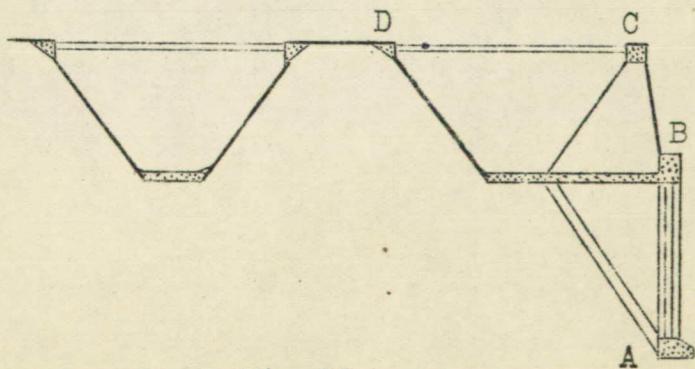


Fig.19

Cross-section of vault at one end of shed.

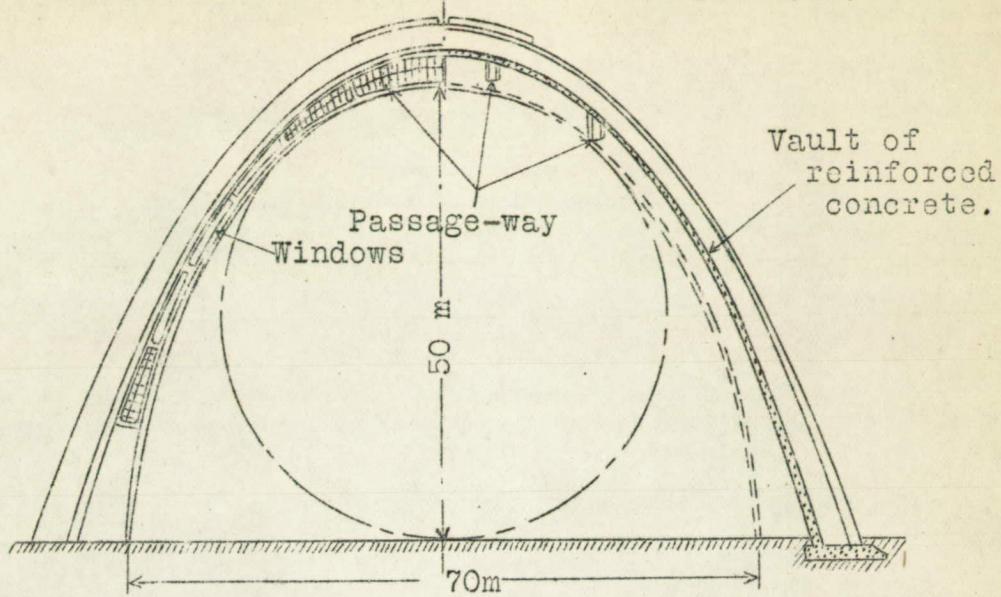
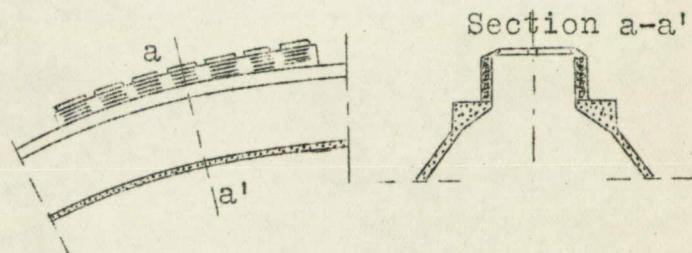


Fig.20 Semi-elevation of shed, showing windows surrounding the doors.



Figs.21 & 22 Skylight and ventilating device at top of shed.

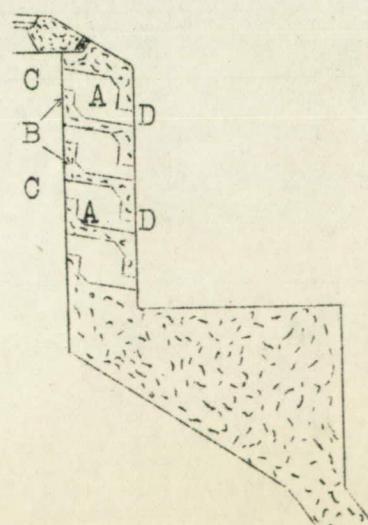


Fig.23 Ventilating device.

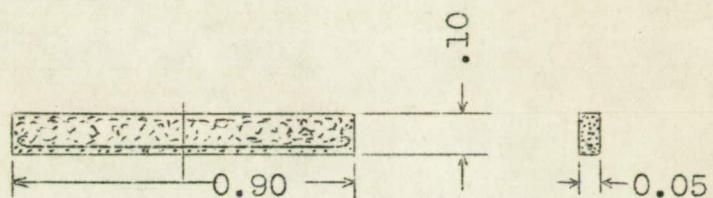


Fig.25 Test-bar of concrete reinforced on only one side.

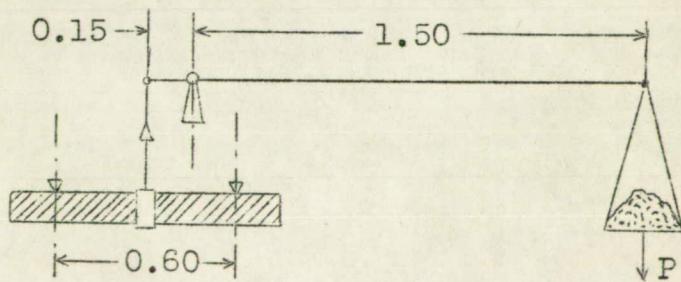


Fig.26 Bending test of above bar.

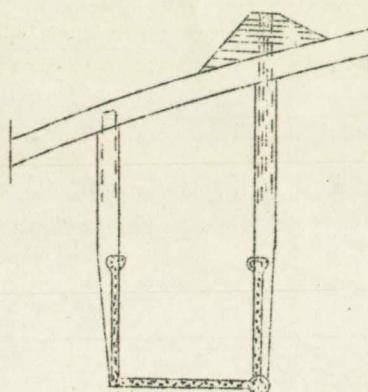


Fig.24
Cross-section of passage-way hung from roof of shed.

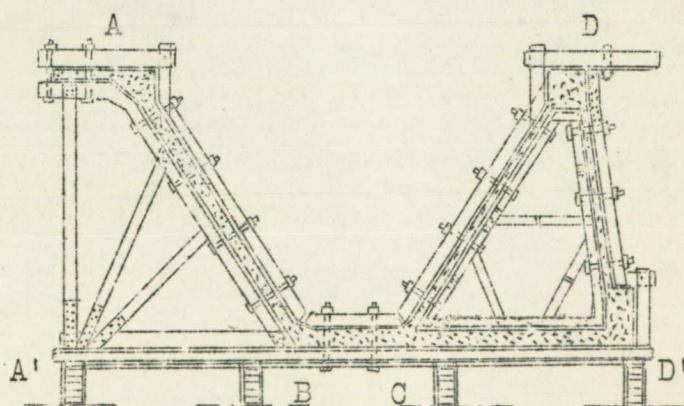


Fig.27
Cross-section of mould showing assembly of elements of wall at one end of shed.